Welding Design and Construction

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(Continued from the November issue)

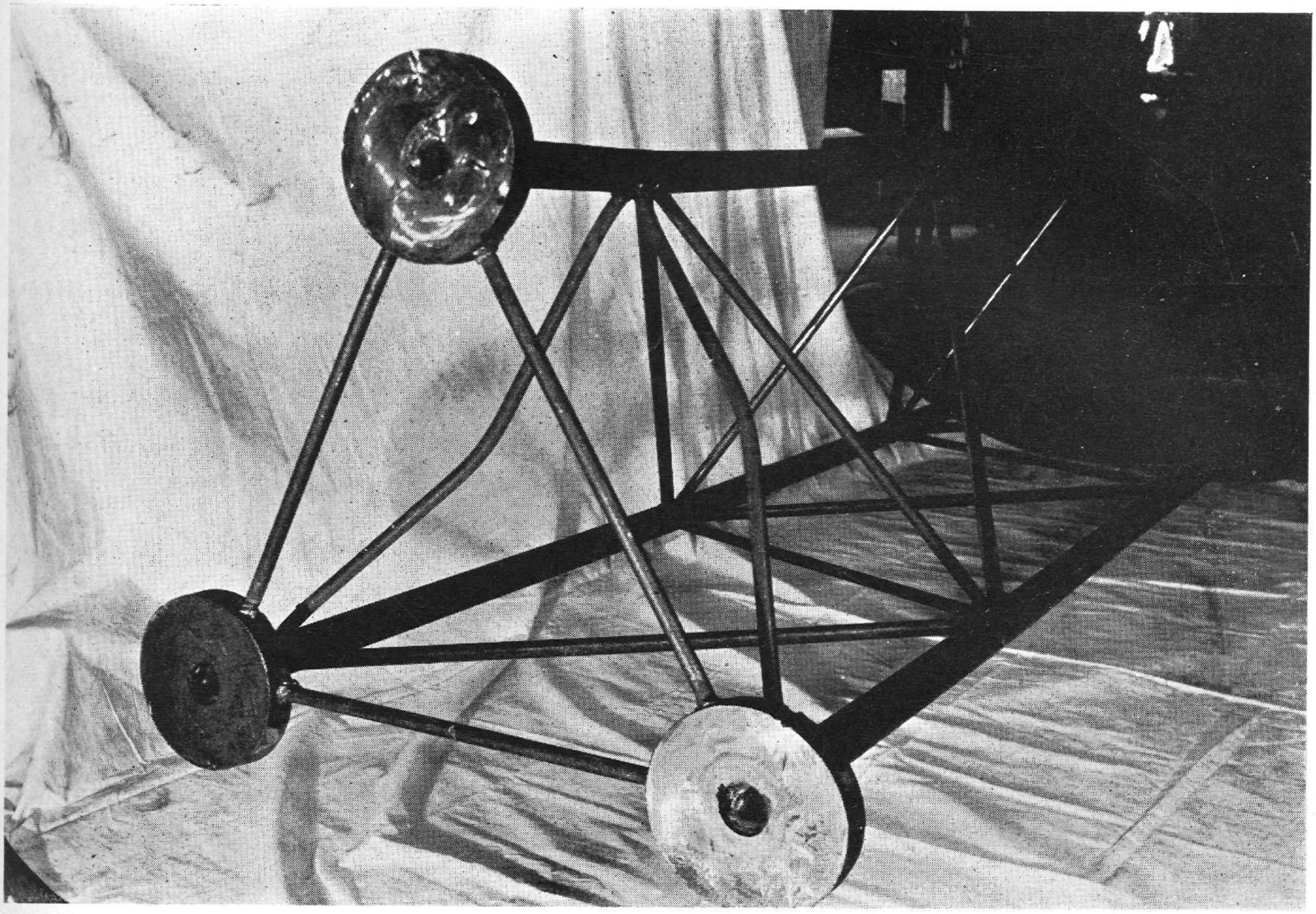

FIGURE 3

Failure of Section with Shelton Web System

Total Length of the Cables When Stress in X is a Maximum

Total length = Initial length + stretch from wind stress

$X = 330.202 + .405 = 330.607$ feet

$Y = 330.202 + .202 = 330.404$ feet

$Z = 330.202 - .256 = 329.946$ feet

Formula (1) is arranged in a slightly different form to make the solution of the deflection of point A possible. Dividing (1) by sec θ gives

$$\frac{L}{\sec\theta} = a + \frac{(8)(a)(\Delta)^2}{(3)(a)^2(\sec^4\theta)}$$

$$(L)x(a)\cos\theta = a^2 + \frac{(8)(\Delta)^2}{3\sec^4\theta}$$

The horizontal distance from the ground attachment of the guy to the vertical projection of point A on the ground now becomes $(a+d)$ as shown in figure 6.

Substituting $(a+d)$ for a and transposing, and solving the quadratic gives the horizontal deflection d.

$$(3)\quad (a+d)^2 - L\cos\theta\,(a+d) + \frac{8\Delta^2}{3\sec^4\theta} = 0$$

The fact that Δ is now actually $= \frac{w(a+d)^2}{8\times H}$ does not however make enough difference to warrant this refinement.

The component of the deflection parallel to guy X is d_x.

$$(a+d_x)^2 - 330.607\cos\theta(a+d_x) + \frac{8(2.23)^2}{3(\sec^4\theta)}$$

Completing the square, $(270+d_x) - 135.188 = 135.166$ $d_x = .354$ feet

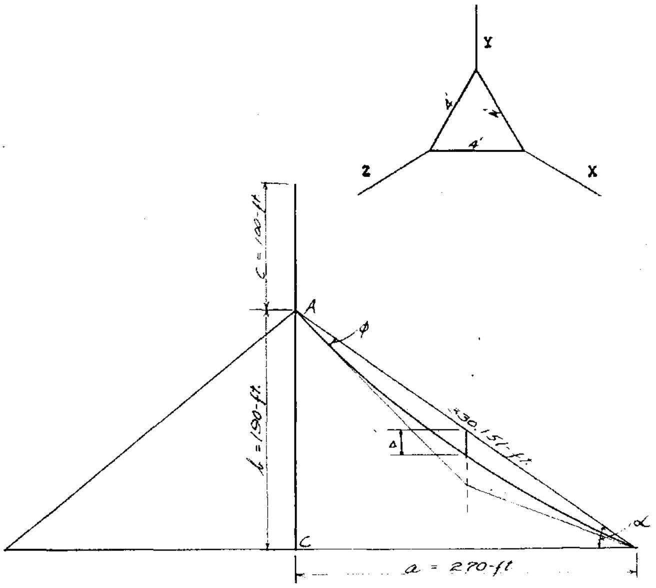

FIGURE 4

Tower Dimensions

The component of the deflection parallel to guy Y is d_y

$$(a+d_y)^2 - 330.404 \cos\theta(a+d_y) + \frac{8(3.56)^2}{3(\sec^4\theta)}$$

$$(270+d_y) - 135.105 = 135.050$$

$$d_y = .155 \text{ feet}$$

$$d_z = -(d_x+d_y) = -(.354+.155 = -.509 \text{ feet}$$

Equation (3) is solved for Δ^2

$$\Delta^2 = \left(\frac{L\cos\theta(a+d) - (a+d^2}{8}\right) 3\sec^4\theta$$

$$(4) \quad \Delta^2 = \frac{3\sec^4\theta}{8}(a+d)\left(L\cos\theta - (a+d)\right)$$

$$\Delta^2_z = \frac{3\sec^4\theta}{8}(270-.509)(329.946\cos\theta - (270-.509))$$

$$\Delta^2_z = \frac{6.711(269.491)(.345)}{8} = 7.80$$

$$\Delta^2_z = 8.82 \text{ feet}$$

$$H_z = \frac{(w)(a)^2}{8\Delta} = \frac{2.32(269.491)^2}{8(8.82)} = 2390 \text{ ft.}$$

$$\frac{192\text{x}12}{54} = 43 \text{ lbs. per foot of tower}$$

Horizontal Component of Initial Stress—H_z = the Horizontal Component of the Relief of Initial Tension.

or 6870—2390=4480 lbs. horiz. comp. of relief of initial tension.

$\frac{4480}{\cos\theta}$ = 5480 lbs. relief of initial tension in guy Z.

5480 lbs. compares very closely with the assumed relief of the initial tension, 5470 lbs., in guy Z.

$$\tan\phi = \frac{(w)(a)}{2\text{x}H_x} = \frac{2.32\text{x}270}{2\text{x}9480} = .0331$$

$$\phi = 1° \ 54'$$

Maximum stress in guy $X = \frac{H_x}{\cos(\theta+\phi)}$

Max $X = \frac{9480}{\cos(35°\ 08' + 1°\ 54')}$ = 11,870 lbs. tension.

The allowable working stress in a high strength ⅞-inch cable is 13,200 lbs. Therefore a ⅞-inch cable is safe for the wind loading specified (25 lbs/sq. ft.).

Stresses in the Mast Above the Guy Attachment

The approximate dead weight per panel of tower (54 inches) is calculated below.

1⅞-inch legs, $\frac{\pi \times 1.875^2\text{x}3\text{x}4.5\text{x}490}{4\text{x}144}$ = 126 lb/panel

1-inch diameter webs, $\frac{\pi\text{x}1^2\text{x}3\text{x}5\text{x}490}{4\text{x}144}$ = 40 lb/panel

⅞-inch diameter webs, $\left(\frac{7}{8}\right)^2 \frac{\pi\text{x}3\text{x}51\frac{1}{4}\text{x}490}{4\text{x}144\text{x}12}$ = 26 lb/panel.

Total 192 lb/panel

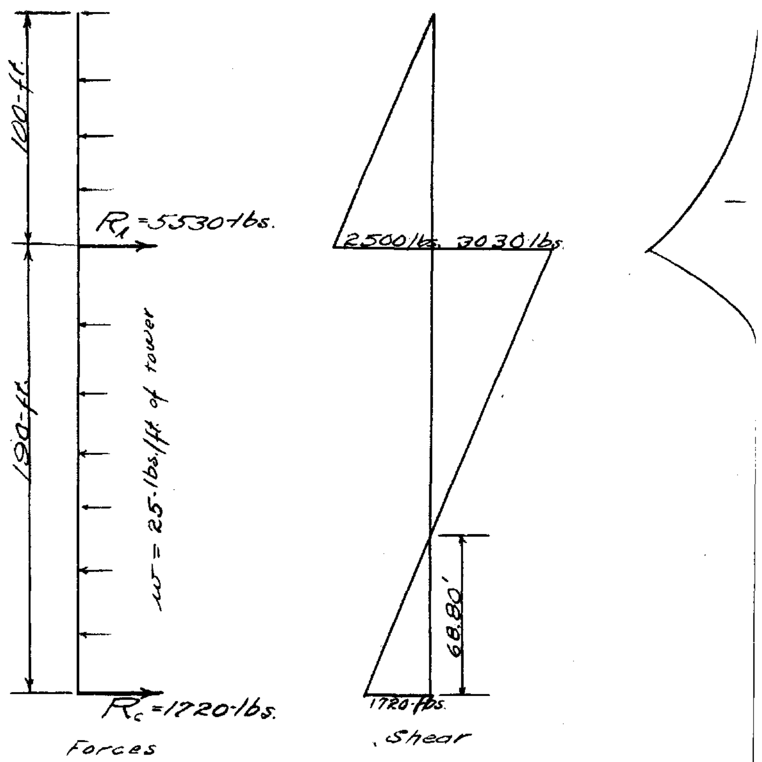

FIGURE 5

Tower Moment and Shear Diagrams

For calculations 45 lbs. of dead weight per foot of tower is used.

The section of the tower above $\bar{A}$, the point of guy attachment, acts as a cantilever.

Moments are taken about point A.

$$3.464F=\frac{(w)\ (1)^2}{2}=\frac{25\,(100)^2}{2}=125{,}000 \text{ ft-lbs.}$$

$$F=\text{Max. comp.}=\text{Max. tension}=\frac{125{,}000}{3.464}=36{,}150 \text{ lbs.}$$

The dead load above point A,

$=45\text{x}100'=4500$ lbs. total.

$\frac{4500}{3}=1500$ lbs. per column member above A.

Maximum compression$=36{,}150+1500=37{,}650$ lbs/column.

Maximum tension$=36{,}150—1500=34{,}650$ lbs/column.

Try a 1⅞-inch round bar, the minimum allowed by $\frac{L}{R}$

$$R=\frac{\text{diameter}}{4}=\frac{1\tfrac{7}{8}}{4}=15/32 \text{ inches}$$

$$\frac{L}{R}=54/15/32=115$$

Allowable compression stress$=10{,}590\text{x}1\tfrac{1}{3}=14{,}110$ lb/sq. in.

Required area$=\frac{37{,}650}{14{,}110}=2.67$ sq. in.

Area furnished by a 1⅞-inch bar$=2.76$ sq. in.

Stresses in the Mast Below the Guy Attachment

The maximum compression occurs in column Z when the wind is blowing. Column Z is subjected to the wind compression, the dead weight, and the pull of guy Z.

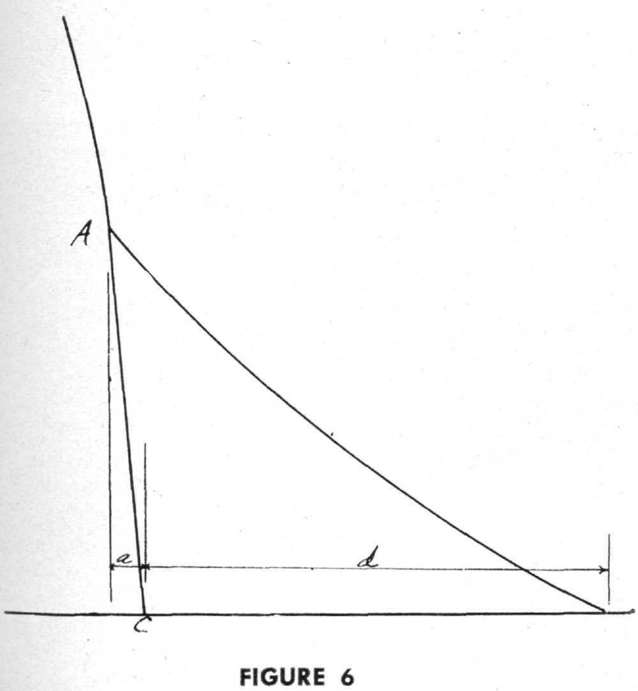

FIGURE 6

Deflected Position of Tower

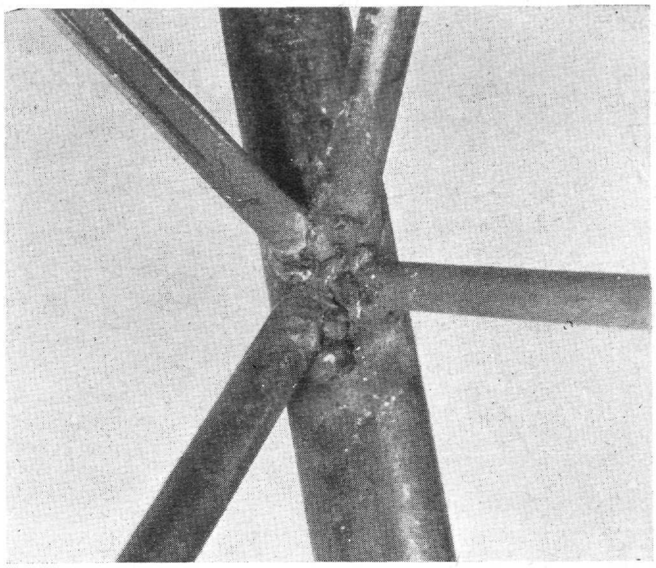

FIGURE 7

Typical Weld Specimen of Shelton Web System

The verticial effect of guy $Z=H_z \text{ x } \tan(\theta+\phi)$

$$\tan\phi=\frac{(w)\ (a)}{2\text{xH}}=\frac{2.32\text{x}270}{2\text{x}2390}=.1311$$

$\phi=7°\ 29'$

$\theta=35°\ 08'$

$42°\ 37'$ $\quad$ $\tan)42°—37'=.92008$

$H_z = 2390$ lbs.

$H_z\text{x} \tan(42°\ 37'=2390\text{x}.92008=2200$ lbs.

Compression due to wind on tower	$=36{,}150$ lbs.
One-third of dead load above A	$=\ \ 1{,}500$ lbs.
$H_z \text{ x } \tan(\theta+\phi)=2390\text{x}.92008$	$=\ \ 2{,}200$ lbs.
Total Compression	$=39{,}850$ lbs.

Required area$=\frac{39{,}850}{14{,}110}=2.82$ sq. in.

Area furnished by a 1⅞-inch round bar$=2.76$ sq. in.

Although a 1⅞-inch bar is stressed slightly above the allowable it will be used here and throughout the tower because the use of a full panel length for L is a conservative measure as was determined in the experimental investigation, even though a full panel length was recommended. Furthermore, a 1⅞-inch bar furnishes adequate cross sectional area at every other critical point in the tower and the use of a 2-inch round bar for the column members would constitute a needless expenditure of steel.

If a given size bar is satisfactory in compression it is not necessary to check it in tension for the maximum tensile stress is equal to the maximum compressive stress minus the dead weight and the allowable tensile stress is $20{,}000\text{x}1\tfrac{1}{3}$lbs/-sq. in. which exceeds considerably the allowable compressive stress.

From the shear diagram, figure 5, the maximum positive moment is seen to occur 68.80 feet above the ground. *(Continued on page 24)*

WELDED RADIO TOWER

(Continued from page 11)

The Design of the Short Web Members

The webs in a given plane receive the maximum stress when the wind blows parallel to the plane of the webs and the shear in the plane is ⅔ of the total wind force. From the shear diagram, figure 5, the maximum shear=3030 lbs.

Maximum shear in the webs=⅔x(3030)=2020 lbs.

Maximum stress in the webs$=\frac{2020\times51\frac{1}{4}}{48}=2160$ lbs. (t) or (c).

Try a ⅞-inch web, the minimum allowed by the $\frac{L}{R}$ ratio.

Area=.601 sq. in. $R=\frac{D}{4}=\frac{7}{4\times8}$

$\frac{L}{R}=\frac{51\frac{1}{4}\times.7}{7/32}=164$

Allowable unit stress=7.22x4/3=9.64 kips/sq. in.

Allowable compression in web=9.64x.601=5.79 kips.

Maximum compression in web=(see above)= 2.16 kips.

The Design of the Long Web Members

Maximum shear in the webs=⅔x(3030)=2020 lbs.

Maximum stress in the webs$=\frac{2020\times60}{48}=2525$ lbs (t) or (C).

Try a 1-inch web, the minimum allowed by the $\frac{L}{R}$ ratio.

Area=.785 sq. in. $R=\frac{D}{4}=\frac{1}{4}$

$\frac{L}{R}=\frac{60\times.7}{\frac{1}{4}}=168$

Allowable unit stress=7.01x4/3=9.35 kips/sq. in.

Allowable compression in web=9.35x.785=7.34 kips

Maximum compression in web (see above)= 2.53 kips

The Design of the Web Weld

For the maximum economy of steel the web weld is designed to develop the full allowable compressive strength of the web members.

The exact perimeter of the web weld cannot be determined because the weld is approximately elliptical in shape as can be seen in figure 7. It is on the safe side to take the length of the weld as the circumference of a circle whose diameter is the diameter of the web plus the distance to the center of gravity of the weld.

A weld is weaker in tension than in compression. Therefore the weld is designed for the greatest tension that is possible in the web without exceeding the allowable compression that is acting simultaneously in the two adjacent webs in the same plane.

The allowable tension or shear through the throat of a fillet weld is 13,600 lbs/sq. in. (A. W. S. Code). It is necessary to find the condition that will give the maximum tensile stress in a web member.

The allowable compression in the long web—$\frac{7.34 \text{ kips}}{\text{web}}$

The simultaneous tension in the adjoining short web is,

$\frac{7.34\times48}{60}\times\frac{51.25}{48}=6.26$ kips (t)

The allowable compression in the short web=$\frac{5.79 \text{ kips}}{\text{web}}$

The simultaneous tension in the adjoining long web is,

$5.79\times\frac{48}{51\frac{1}{4}}\times\frac{60}{48}=6.78$ kips web.

A ¼-inch fillet weld is recommended as the minimum size weld for this type of outdoor structure.

A ¼-inch weld is tried for the short web—diameter ⅞ inch.

The allowable tension per inch of weld,
.707x13,600x¼=2400 lbs.

The diameter to center of gravity of the weld,
(⅞+⅓x¼+⅓x¼)=1.042 inches.

Circumference of the weld=(1.042)π=3.28 inches.

Allowable tension on the weld,
3.28x2400=7.88 kips in short web.

Max. tension on the weld=6.26 kips in short web.

A ¼-inch weld is tried for the long web—diameter 1 inch.

The allowable tension per inch of weld,
.707x13,600x¼=2400 lbs.

The diameter to center of gravity of the weld,
(1+⅓x¼+⅓x¼)=1.167 inches.

Circumference of the weld=(1.167)π=3.66 inches.

Allowable tension on the weld,
3.66x2400=8.79 kips in long web.

Max. tension on the weld=6.78 Kips in long web.

Design of the Connection Plate Weld

The connection plate weld is designed to develop the full tensile strength of the column.

The maximum tension that occurs in the tower leg is 34,650 lbs. The weld is designed for the

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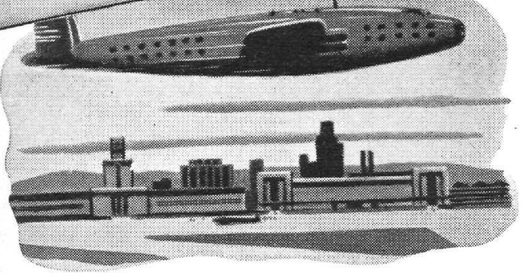
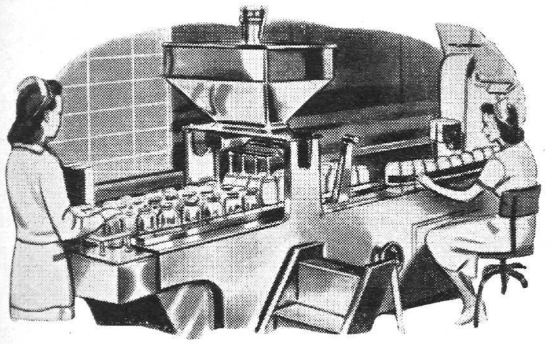
Filling Prescriptions with Steel!

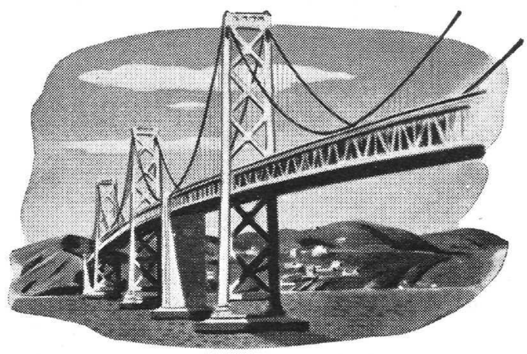


1. BETTER STEELS! This country needs aircraft that fly high and far . . . and hit hard. It needs ships in great numbers. It needs tanks that can take it when the going gets tough. It needs equipment to outperform any on earth. All these things require many special steels. Such steels with needed properties are created through the use of *alloys*. Basic peacetime research by ELECTRO METALLURGICAL COMPANY, a Unit of UCC, has developed many important steels and the alloys to make them, such as chromium, silicon, manganese, vanadium, tungsten, calcium, and columbium . . . all vital today.



2. STAINLESS STEELS! The development of steels of high chromium content gave designers and engineers a whole family of new materials with which to work. Such steels resist rust and corrosion, and are easily kept clean. They are essential in the food industry. Possessing great strength in addition to their corrosion resistance, they save weight in trains and planes. They have brought improvements in the oil, chemical, textile, and other fields...with resultant savings to you. Low-carbon ferrochromium, an Electromet development, is essential in the large-scale production of stainless steels.

3. NEW NATIONAL RESOURCES! Tungsten and vanadium are essential to steelmakers. Long before war clouds loomed, many felt that more of this country's domestic sources of these metals should be developed. Engineering research by UNITED STATES VANADIUM CORPORATION, another UCC Unit, found efficient ways of refining low-grade ores. This enabled U. S. VANADIUM to revitalize old mines with new mills and methods, and make America less dependent on foreign sources for her increased needs of tungsten and vanadium.



4. BUILDING TOWARDS THE FUTURE! Alloy steels offer still greater promise for the future. Bridges and other structures will be made still lighter, stronger, and longer-lasting by wider use of some of the steels with which engineers are already experienced. Trains, trucks, and aircraft will be made lighter, stronger, faster, and safer. Better cars and tractors, homes and home equipment will be made through their use.

Units of UCC do not make steel. They do make ferro-alloys used to purify and give special properties to steel. They also make non-ferrous alloys which, because of their exceptional resistance to wear, heat, and corrosion, are used as cutting tools, hard-facing welding rods, and for other purposes. UCC research and developments mean ever-new and improved alloys for industry . . . and ever-better products for you.

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WELDED RADIO TOWER

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maximum allowable tension which is 20,000x2.76 = 55,200 lbs.

A $\frac{7}{8}$-inch fillet weld is tried. The column diameter is $1\frac{7}{8}$ inches. See figure 7.

The allowable tension per inch of weld,

$(.707 \times 13{,}600 \times \frac{7}{8}) = 8400$ lbs.

The diameter to the center of gravity of the weld,

$(1\frac{7}{8} + \frac{2}{3} \times \frac{7}{8}) = 2.458$ inches.

Circumference of the weld $= (2.458)\pi = 7.73$ inches.

Allowable tension on the weld,

$7.73 \times 8400 = 64{,}900$ lbs. tension.

Allowable tension in the column = 55,200 lbs.

A $\frac{7}{8}$-inch weld is therefore sufficient to develop the full tensile strength of the column and transfer this tension to the connection plate.

Design the bolts to develop the full strength of the tower column or 55,200 lbs. tension.

$$\frac{55{,}200}{(20{,}000)\frac{4}{3}} = 2.07 \text{ sq. in. required root area.}$$

Four 1-inch bolts with .551 root area = 2.204 sq. in.

Five $\frac{7}{8}$-inch bolts with .419 root area = 2.095 sq. in.

Seven $\frac{3}{4}$-inch bolts with .302 root area = 2.114 sq. in.

Use five $\frac{7}{8}$-inch bolts with hex heads and hex nuts. The long dimension of a hex is $1\frac{1}{2}$ inches. A 7-inch connection plate will be required to provide clearance distance between the nut and the weld.

Design of the Connection Plate Thickness

The exact behavior of two connection plates bolted rigidly together around the edges and pulled apart at the centers defies exact analytical solution. The bending condition of the plate apparently varies somewhere between full fixity, and a free end condition. The bending moment is taken as the product of the tension in one bolt and the arm measured from the centerline of the bolt to the face of the column member. The bending moment for full fixity is one-half the moment for the free end condition.

For a working moment the average of the free and the fixed end moment is taken.

$$\text{Tension per bolt} = \frac{55{,}200}{5} = 11{,}040 \text{ lbs.}$$

$$\text{Moment arm} = 2\tfrac{5}{8} - \frac{1\frac{7}{8}}{2} = 1\ 9/16 = 1.562 \text{ inches}$$

Free end moment $= 1.562 \times 11{,}040 = 17{,}245$ in. lbs.

$$\text{Fixed end moment} = \frac{1.562 \times 11{,}040}{2} = \underline{8{,}622} \text{ in. lbs.}$$

$2/\overline{25{,}867}$ in. lbs.

Average of Moments = 12,934 in. lbs.

$$\text{From mechanics, Stress} = \frac{\text{Moment}}{\text{Section Modulus}} \text{ or } = \frac{M}{Z}$$

$$\text{Required } Z = \frac{M}{S} = \frac{12{,}934}{20{,}000 \times 4/3} = .485 \text{ in.}^3$$

If b = width, d = depth, and I = Inertia then

$$Z = \frac{I}{b/2} = \frac{(b)(d)^3}{12/d/2} = \frac{(b)(d)^3}{6}$$

$$d^2 = \frac{6xZ}{b}$$

It is apparent that the maximum bending stress occurs at the face of the column, but the question arises as to what value of b to use. The ideal condition would exist when the stress from each bolt spread out evenly over that fraction of the connection plate represented by that bolt, in this case 1/5. It is also apparent that increasing the number of bolts will aid in more nearly reaching the ideal condition. It will be assumed that five bolts are sufficient to evenly distribute the moment to the face of the column. b will therefore be 1/5 of the circumference of the column leg.

$$b = \frac{D\pi}{5} = \frac{1.875\pi}{5} = 1.178 \text{ inches.}$$

$$d^2 = \frac{6x.485}{1.178} = 2.47$$

d = 1.57 inches.

Use a plate 7 inches in diameter by $1\frac{5}{8}$ inches thick.

It is now evident that the design of a welded steel tower is relatively straight forward mechanics of a not too complicated nature.

It is also evident that the use of welding cuts field work to an absolute minimum and relegates all of the construction work to the shop where the tools of construction are more readily available. The standardization of each section of the tower lends to simplicity of fabrication.

The welded joints give a permanent rigidity and fixity to the tower that is not obtainable with the bolted type of tower.

The Shelton web system, used in this design, does not reduce the L (free length) of the column members, but the twisting and bending in the columns is definitely less than in a similar tower having the conventional Warren web system, a finding that is borne out by the results of extensive testing of the two types of towers.